Mahendran, Mahen
(2001)
Design of steel roof and wall cladding systems for pull-out failures.
Steel Construction, *35*(1), pp. 14-27.

DESIGN OF STEEL ROOF AND WALL CLADDING SYSTEMS FOR PULL-OUT FAILURES

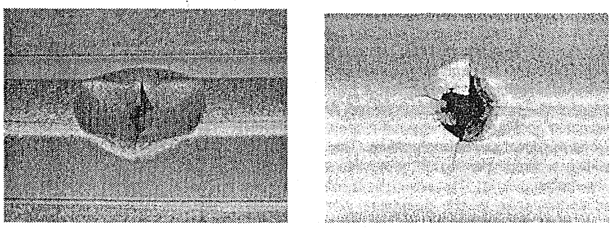
M. Mahendran

Physical Infrastructure Centre, School of Civil Engineering
Queensland University of Technology, Brisbane

Abstract: When thin steel roof and wall cladding systems are subjected to wind uplift/suction forces, local pull-through or pull-out failures occur prematurely at their screwed connections. During high wind events such as storms and cyclones, these localized failures then lead to severe damage to buildings and their contents. In recent times, the use of thin steel battens, purlins and girts has increased considerably, which has made the pull-out failures more critical in the design of steel cladding systems. An experimental investigation was therefore carried out to study the pull-out failure using both static and cyclic tests for a range of commonly used screw fasteners and steel battens, purlins and girts. This paper presents the details of this experimental investigation and its results.

1. Introduction

Extreme wind events such as cyclones and storms often cause severe damage to large number of low-rise buildings. Damage investigations following these extreme wind events have always shown that disengagement of steel roof and wall cladding systems has occurred due to local failures of their screwed connections under wind uplift or suction loading (see Figures 1 and 2). The steel sheeting is made of thin high strength steels (G550 steel: 0.42 mm base metal thickness and minimum yield stress 550 MPa) and is intermittently crest-fixed. Such profiled steel sheeting often pulls-through the screw heads (Figure 1(a)) due to the large stress concentration around the fastener holes under wind uplift/suction loading [1].



(a) Static (b) Fatigue

Figure 1. Pull-through Failures

Sustained fluctuations of wind uplift loading during a cyclone have been shown to cause fatigue cracking in this steel sheeting around the fastener holes at rather lower load levels [2,3]. This also leads to a pull-through failure as shown in Figure 1 (b). Both static and fatigue type pull-through failures lead to rapid disengagement of all roof and wall claddings, causing severe damage to the entire building. The local pull-through failure phenomenon has been investigated in detail by many researchers in the past [1-5].

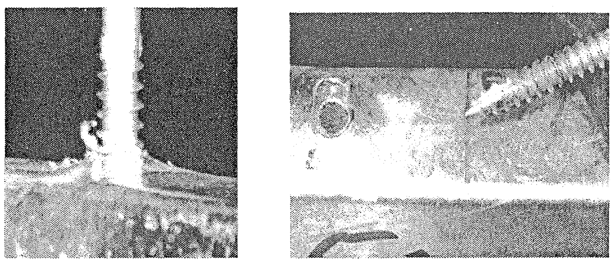


Figure 2. Pull-out Failure

In recent times, very thin high-strength steel battens of various shapes have been used in housing, industrial and commercial buildings and this appears to be the fastest growing method in roof construction. These cladding systems can then suffer from another type of local failure when the screw fasteners pull-out of the steel battens, purlins or girts (see Figure 2). Such a pull-out failure also leads to a rapid disengagement of roof and wall claddings, causing severe damage to the entire building. Therefore an experimental investigation was conducted to investigate the static and fatigue pull-out behaviour of these steel cladding systems under static and cyclic wind uplift/suction load conditions for a range of commonly used screw fasteners and steel purlins, battens and girts. The applicability of the general design formula for static pull-out strength to roof and wall cladding systems was investigated first. An improved formula was then developed in terms of the thickness and ultimate tensile strength of steel and thread diameter and pitch of screw fasteners under static wind uplift load conditions. Cyclic tests were used to investigate the possible strength reduction due to sustained fluctuating wind loading conditions during storms and cyclones. This paper presents the details of this investigation and its results.

2. Current Design and Test Methods

The Australian [6], American [7] and the European provisions [8] include design formulae for the pull-out capacity, F_{ou}, of screw connections in tension as shown by Equations (1a) and (1b).

Australian and American $F_{ou} = 0.85\ t\ d\ f_u$ (1a)
European $F_{ou} = 0.65\ t\ d\ f_y$ (1b)
where t = thickness of member, d = nominal screw diameter, f_u = ultimate tensile strength of steel and f_y = yield stress of steel.

The design pull-out capacity is obtained by using a capacity reduction factor of 0.5 to Equations (1a) and (1b). Pekoz [9] and Toma et al. [10] present the background to the American and European equations, respectively. The difference between these equations is partly due to the European equation being based on a characteristic strength (5 percentile) whereas the American equation is based on an average strength. These design equations were developed for conventional fasteners and thicker mild steel. At present, the American and Australian codes recommend the use of 75% of the specified minimum strength for high strength steels such as G550 steel with a yield stress greater than 550 MPa and thickness less than 0.9 mm to allow for the reduced ductility of these steels. Since the design formulae are considered to be conservative, the design for the pull-out failure of screwed connections in tension is mainly based on laboratory experiments.

In the past, different test methods such as the U-tension, cross tension and plate methods have been used for testing screw connections in tension. The American and European specifications [7,8] are based on the U-tension method whereas the Australian provision [6] recommends the cross-tension method. The background to these test methods is given in Macindoe and Hanks [11]. Macindoe et al. [12] have used the cross tension test method to review the applicability of American design formula given by Equation (1a) for thin high strength steels such as G550 steels. Based on this, Macindoe et al. [12] modified the predictive equations for pull-out strength, F_{ou} (Equation (2)). It includes the term $f_u^{\ 0.5}$ to eliminate the need for the use of 75% of the specified minimum strength for G550 steels with thickness less than 0.9 mm. But their work is not specific to roof and wall cladding systems.

$$F_{ou} = 35\sqrt{(t^{2.2} d f_u)} \qquad (2)$$

where t, d and f_u are as defined for Equation (1a).

3. Experimental Investigations

3.1 Static Tests

Since the main aim of this investigation was to develop specific design information for the pull-out strength of steel roof and wall cladding systems, the more general standard cross tension test method was not used, but instead conventional two-span cladding tests and appropriate small scale batten/purlin tests were conducted to better simulate the realistic behaviour of steel roof and wall cladding systems.

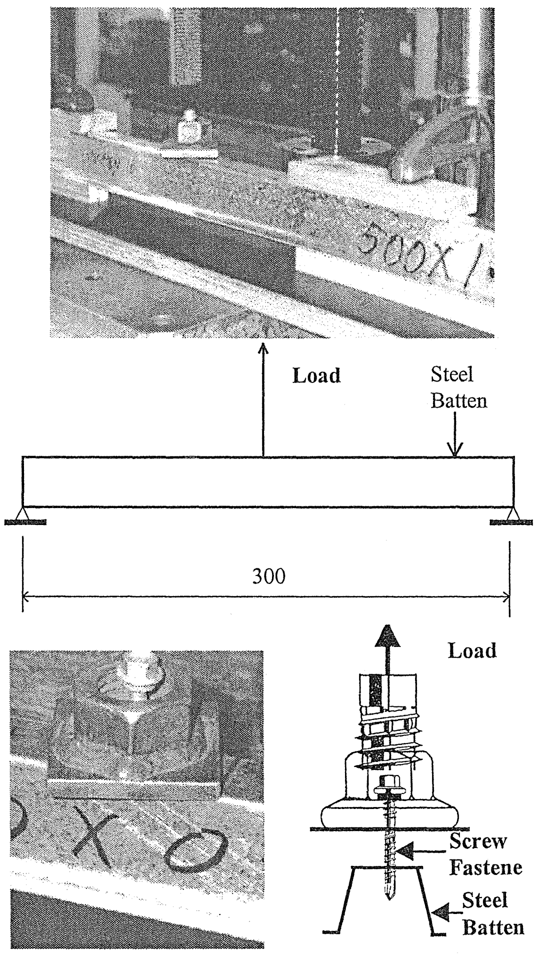


Figure 3. Static Test Set-up

Since the pull-out failures are localized around the screw holes on the batten/purlin (see Figure 2), a small scale test method was used to simulate this failure. A batten supported at shorter spans with only one or four screw fasteners was used with tension force being applied to the fastener head. Test results showed that the difference in pull-out failure loads between the two-span cladding test method, the multiple screw fastener method and the single screw fastener method was insignificant [13]. It was also found that test span in the single screw fastener method did not cause any changes to the failure load. Therefore the single screw fastener method with a span of 300 mm was used in this investigation. Figure 3 shows the chosen test method. It was considered that this method would simulate the local flexing of the steel batten around the fastener hole and the appropriate tension loading in the screw fastener to produce the pull-out failure load one would obtain by testing a two-span cladding. This test method is very simple to use and enables a large number of pull-out tests to be completed with limited resources in a short period of time.

Table 1. Details of Steel Battens and Purlins

Steel Grades	BMT (mm)		Yield Stress f_y (MPa)		Ultimate Stress f_u (MPa)	
	Nominal	Measured	Nominal	Measured	Nominal	Measured
	0.40	0.38		358		415
G250, Battens	0.60	0.54	250	359	320	399
	1.00	0.95		332		390
	0.42	0.43		717		721
G550, Battens	0.60	0.61	550	696	550	703
	0.95	0.95		639		655
G500, Battens	1.20	1.20	500	635	520	647
G450, Battens	1.60	1.58	450	584	480	604
	1.90	1.79		497		560
G450, Purlins	2.40	2.30	450	465	480	587
	3.00	2.93		450		553

Table 2. Details of Screw Fasteners

Screw Type	Gauge	Thread Diameter d (mm)		Thread Form (threads per inch)	Thread Pitch p (mm)
		Nominal	Measured		
HiTeks	10-16	4.87	4.67	16	1.59
	10-24	4.87	4.67	24	1.06
	12-11	5.43	5.52	11	2.31
	12-14	5.43	5.47	14	1.81
	12-24	5.43	5.36	24	1.06
	14-10	6.41	6.39	10	2.54
	14-20	6.41	6.22	20	1.27
Type 17	10-12	4.87	4.81	12	2.12
	12-11	5.43	5.53	11	2.31
	14-10	6.41	6.34	10	2.54
Series 500	12-24	5.43	5.49	24	1.06

Following the validation of the single screw fastener test method, a series of pull-out tests was conducted for a range of steel battens, purlins or girts and screw fasteners, which are commonly used in the building industry. The steel battens, purlins/girts covered a range of different thicknesses from 0.4 mm to 3.0 mm BMT, and steel grades from G250 to G550 (minimum yield stress from 250 to 550 MPa). The screw fasteners covered a range of different screw gauges from 10 to 14 (nominal thread diameter d from 4.87 to 6.41 mm), and thread form from 10 to 24 threads per inch (thread pitch p from 2.54 to 1.06 mm). Tables 1 and 2 give the details of steel battens and purlins and screw fasteners used in this investigation, respectively. Five tests were conducted for a combination of each batten/purlin in Table 1 and each type of screw fastener in Table 2, resulting in a total of 592 tests. A total of 55 standard tensile tests were also conducted to determine the tensile strength properties (yield and ultimate stresses) of the steel used in steel battens. The measured and specified (nominal) tensile strength values are given in Table 1.

A preliminary series of tests on battens with different geometry showed that the batten geometry has very little effect on pull-out strength.

Hence a batten geometry that is commonly used in the building industry was chosen. For the tests on thicker purlins, available purlins of three different sizes were used. Figure 4 shows the geometry of battens and purlins used in this investigation.

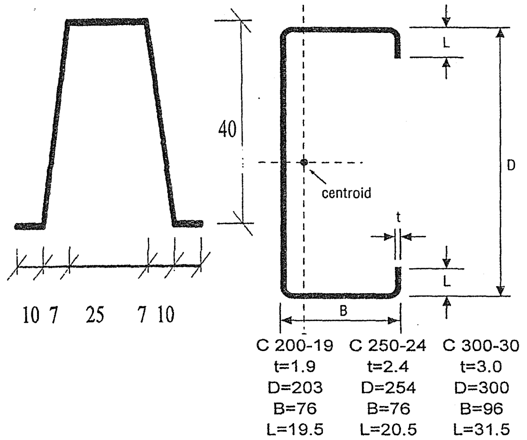


Figure 4. Test Battens and Purlins

As seen in Table 2, screw fasteners with three different drill points, namely, HiTeks, Type 17 and Series 500 (see Figure 5), were chosen [14]. HiTeks screws are used in fixing to metal battens, purlins or girts of less than 6.0 mm thickness whereas Type 17 screws are used in fixing to timber purlins. However, the latter is commonly used in the building industry for thin battens of less than 1.0 mm thickness. Therefore Type 17 screws were also included in this investigation. Series 500 Teks screws are used mainly for thicker metal purlins up to 12.0 mm thickness, however, they were also included. For each type of screw fastener, the thread diameter and thread form were varied as shown in Table 2.

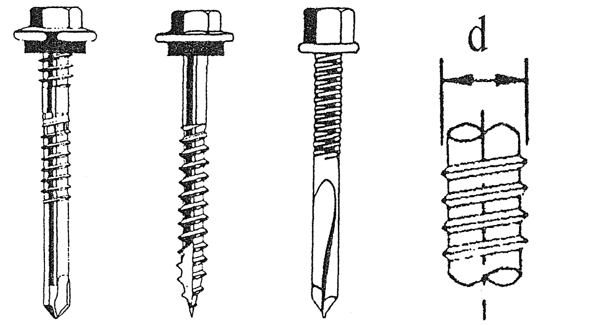


Figure 5. Screw Fasteners

The test specimens were loaded at a loading rate in the range of 3-5 mm/minute until the screw fasteners pulled-out of the battens/purlins.

3.2 Static Test Results and Discussion

3.2.1 Results

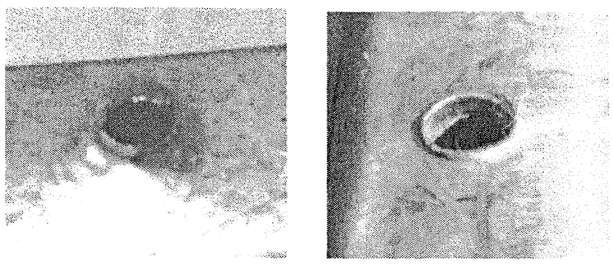
Table 3 presents typical pull-out failure loads for one HiTeks screw fastener. Other results are presented in [15]. The results were grouped based on thickness and grade of steel, analysed and comparisons made based on these groups.

Table 3. Experimental Results for 10-24×25 HiTeks Screw Fasteners

Thickness t (mm)	Steel Grade	Failure Load (N/fastener) Experimental Records (N/f)	Mean (N/f)	Std. Dev.
0.40	G250	475, 345, 400, 418, 445	417	42
0.60		548, 578, 643, 603, 593	593	28
1.00		1343, 1315, 1323, 1365, 1370	1343	28
0.42	G550	715, 758, 793, 648, 743, 755, 815	746	58
0.60		930, 918, 1030, 990, 890	952	64
0.95		2100, 1890, 2100, 2100, 2120	2062	109
1.20	G500	2650, 2720, 2790, 2190, 2440	2558	275
1.60	G450	3610, 3290, 3560, 3100, 3980	3508	382
1.90		4750, 4610, 4600, 4870, 4660	4698	126
2.40		7150, 6720, 6000, 6450, 6670	6598	328
3.00		8650, 9010, 8930, 8900, 9370	8972	217

Note: Std. Dev. = Standard Deviation

Two different types of pull-out failure modes were observed. In thin steels, for which the thickness is less than the thread pitch, the steel batten around the screw hole was bent as the screw threads were withdrawn. In thicker steels, where the thickness is greater than the thread pitch, the steel batten/purlin around the screw hole was sheared off as the screw threads were withdrawn. Figure 6 shows these two pull-out failure modes.



(a) Mode 1 (b) Mode 2

Figure 6. Static Pull-out Failure Modes

In general, it was found that Type 17 screw fasteners gave a higher pull-out load compared with other screw fasteners of the same size. This implies that the type of thread and drill point can influence the pull-out strength. However, this aspect was not investigated in detail.

3.2.2 Comparison of Test to Predicted Values Based on Current Design Formula

The pull-out failure load results from tests (see Table 3) were compared with the predictions from the current design formula given by Equation (1a) using both the measured and specified (nominal) values for the properties of the steel and screw fasteners. Table 4 presents the comparisons for each grade and thickness of steel and groups of screw fasteners using the measured properties (Case 1: All, Case 2: HiTeks + Type 17, Case 3: HiTeks, Case 4: Type 17).

Table 4. Test to Predicted Values Based on Current Design Formula and Measured Properties

Steel		Case 1		Case 2		Case 3		Case 4	
Grade	Thickness	Mean	COV	Mean	COV	Mean	COV	Mean	COV
G250	0.40	0.82	0.21	0.84	0.18	0.76	0.13	1.03	0.06
G250	0.60	0.83	0.21	0.85	0.20	0.75	0.11	1.08	0.06
G250	1.00	0.98	0.13	1.00	0.12	0.95	0.11	1.12	0.05
G250	**t≤1.00**	**0.88**	**0.20**	**0.90**	**0.18**	**0.83**	**0.16**	**1.07**	**0.06**
G550	0.42	0.67	0.17	0.69	0.16	0.62	0.11	0.82	0.05
G550	0.60	0.65	0.17	0.66	0.16	0.61	0.15	0.77	0.05
G550	0.95	0.94	0.19	0.96	0.19	0.91	0.21	1.08	0.04
G500	1.20	0.93	0.10	0.93	0.10	0.87	0.08	1.03	0.03
G550+G500	**t≤1.20**	**0.78**	**0.24**	**0.80**	**0.23**	**0.74**	**0.24**	**0.91**	**0.15**
G450	1.60	1.09	0.12	1.10	0.13	1.05	0.11	1.24	0.08
G450	1.90	1.14	0.12	1.16	0.11	1.12	0.10	1.25	0.12
G450	2.40	1.29	0.13	1.31	0.12	1.28	0.10	1.39	0.16
G450	3.00	1.32	0.08	1.33	0.08	1.34	0.07	1.30	0.09
G450	**1.6≤ t ≤3.0**	**1.21**	**0.14**	**1.22**	**0.14**	**1.20**	**0.14**	**1.28**	**0.12**
G450+G500+G550	**t≤3.0**	**0.99**	**0.28**	**1.00**	**0.28**	**0.97**	**0.30**	**1.08**	**0.22**
G250 to G550	**t<1.50**	**0.82**	**0.23**	**0.83**	**0.23**	**0.78**	**0.22**	**0.98**	**0.14**
G250toG550	**t≤3.00**	**0.96**	**0.27**	**1.00**	**0.26**	**0.93**	**0.28**	**1.08**	**0.19**

Note:
1. Case 1 = All (HiTeks+Type17+Series500); Case 2 = HiTeks+Type17; Case 3 = HiTeks; Case 4 = Type17
2. COV = Coefficient of Variation

As seen in Table 4 results, the mean Test to Predicted values are less than 1.0 for all cases except for the thicker G450 steel, which reveals the inadequacy of the current design formula. The current design formula is less conservative for the thinner G500+G550 steels than for G250 steel for all types of screw fasteners (Cases 1 to 4). However, for the thicker G450 steel, the formula appears to be very conservative. The mean Test to Predicted value is lower for all grades of thinner steel. These observations imply that the current design formula is conservative only for thicker and softer grade steels, and agree well with Macindoe et al.'s [12] observations. It may be unsafe to use the design formula for thinner steels less than 1.5 mm, in particular for G550 steel.

By comparing the results in Table 4 with Macindoe et al.'s [12] results obtained using the general test method of cross-tension specimens, it was found that Macindoe et al.'s results gave higher mean Test to Predicted values in all cases; for example, Macindoe et al.'s results gave a mean value of 1.27 for G250 steels and Case 1 screw fasteners compared with 0.88 in this investigation. This implies that the general test method of using cross-tension specimens could have produced unconservative results.

As seen in Table 4, the type of screw fastener has not caused any significant difference in results. The Type 17 screw (Case 4) is the only one, which appears to provide slightly higher mean Test to Predicted values. This may be because of the higher pull-out loads obtained for Type 17 screw fasteners. Therefore in the discussion of results, only the case of all screw fasteners (Case 1) was considered. No attempt was made to develop separate formulae for the three screw fastener types used in this investigation. However, it must be noted that they are all self-drilling screws.

When specified properties were used, the mean values increased to more than 1.0 for all cases [13]. The use of 75% of specified tensile strength for G550 steel less than 0.9 mm has caused the mean Test to Predicted value for G550+G500 steels (1.14) to be greater than that of G250 steel (1.02). Therefore the use of current design formula with specified properties is preferred, and appears to be capable of predicting the pull-out strengths. These observations are similar to those made by Macindoe et al. [12].

3.2.3 Comparison of Test to Predicted Values Based on a New Design Formula

Mahendran and Tang [13,15] present the details of comparisons when Macindoe et al.'s modified formula (Equation (2)) was used. Although the modified formula appears to better model the pull-out strength than the current design formula (Equation (1a)), specific design formulae were developed for the pull-out failure in the battens and purlins/girts commonly used in the building industry. In order to find the more accurate equation for the pull-out strength F_{ou} of steel roof and wall cladding systems, all the parameters on which the strength is dependent were included in the analysis. Therefore the thread diameter d and thread pitch p of the screw fastener and base metal thickness t and tensile strength f_u of the batten/purlin material were all included in the new design formula given by Equation (3). The use of ultimate tensile strength f_u gave a better correlation between the actual and predicted results than the yield strength f_y. Therefore, f_u was used in Equation (3). When compared with Equations (1) and (2), the new equation includes an additional parameter, the thread pitch p, as it was often found to affect the pull-out capacity.

$$F_{ou} = k \ d^m \ p^n \ t^v \ f_u^w \tag{3}$$

where k, m, n, v and w are constants

The unknown constants k, m, n, v and w were determined using the "Solver" in Microsoft Excel which is based on the method of least squares. Separate equations were derived for different groups as shown in Table 5. Although equations were derived for each group of screw fasteners (Cases 1 to 4), only Case 1 with all screw fasteners was considered in the final analysis as there was little difference between the different types of screw fasteners. For the derived equations, the mean Test to Predicted values and coefficient of variation (COV) were also calculated and are included in Table 5.

Table 5. Test to Predicted Values Using the New Design Formula and Measured Properties for Case 1 Screw Fasteners

Steel		Coefficients					**Simplified Formula**		Current Formula		Modified Formula	
Grade	Thickness	k	m	n	v	w	**Mean**	**COV**	Mean	COV	Mean	COV
G250	t<1.5	1.40	0.6	0.3	1.2	1.0	**0.99**	**0.13**	0.88	0.20	1.05	0.20
G550+G500	t<1.5	0.95	0.8	0.2	1.4	1.0	**1.00**	**0.17**	0.78	0.24	1.19	0.21
G450	1.5< t≤3.0	0.90	0.9	0.2	1.3	1.0	**0.98**	**0.10**	1.21	0.14	1.53	0.14
G450+G500+G550 t≤3.0		0.80	0.9	0.2	1.4	1.0	**1.02**	**0.14**	0.99	0.28	1.36	0.21
G250toG550	t<1.5	1.30	1.0	0.2	1.3	0.9	**1.02**	**0.18**	0.82	0.23	1.13	0.21
G250toG550	t≤3.0	0.80	0.9	0.2	1.4	1.0	**1.07**	**0.16**	0.96	0.27	1.27	0.24

In Table 5, Test to Predicted values using Equations (1a) and (2) (current design formula and Macindoe et al.'s modified formula) are also included for comparisons with the corresponding values from the new formula. The new formulae with appropriate values for the parameters k, m, n, v and w in Equation (3) appear to provide improved mean (closer to 1.0) and coefficient of variation values (COV less than 0.2) in all cases. However, in order to reduce this to a single equation for all groups, the parameters m, n, v and w were forced to be 1.0, 0.2, 1.3 and 1.0, respectively. The values of k were changed to get the best agreement with test results. This is considered acceptable as the coefficients of variation values (COV) are still within 0.18 (see Table 6)

.

Table 6. Test to Predicted Values Using the New Simplified Design Formula

Steel		Coefficients					Measured Properties			Specified Properties		
Grade	Thickness	k	m	n	v	w	Mean	COV	Φ	Mean	COV	Φ
G250	t<1.5	0.75	1.0	0.2	1.3	1.0	1.04	0.15	0.61	1.19	0.17	0.54
G500+G550	t<1.5	0.70	1.0	0.2	1.3	1.0	0.94	0.16	0.53	1.19	0.16	0.54
G450	1.5< t≤3.0	0.80	1.0	0.2	1.3	1.0	0.93	0.10	0.59	1.06	0.11	0.55
G450+G500+G550 t≤3.0		0.75	1.0	0.2	1.3	1.0	0.93	0.15	0.55	1.12	0.13	0.55
G250toG550	t<1.5	0.70	1.0	0.2	1.3	1.0	1.02	0.18	0.56	1.22	0.17	0.55
G250toG550	t≤3.0	0.75	1.0	0.2	1.3	1.0	0.96	0.16	0.56	1.14	0.15	0.54

Note: COV = Coefficient of Variation

Table 6 presents the Test to Predicted values based on these changes to the above parameters for Case 1 (all screw fasteners). These equations are much simpler and at the same time they are quite satisfactory as the mean and coefficient of variation values are similar to those in Table 5 and are acceptable. Therefore the following simplified formula is recommended:

$$F_{ou} = k \; d \; p^{0.2} \; t^{1.3} \; f_u \qquad (4)$$

where $k = 0.70$ for thinner steel battens made of G250, G500 and G550 steel of thickness $t < 1.5$ mm, $k = 0.80$ for thicker steel purlins and girts made of G450 steel of thickness $1.5 < t \leq 3.0$ mm, and $k = 0.75$ for all steel battens and purlins/girts made of G250, G450, G500 and G550 steel of thickness $t \leq 3.0$ mm. It must be noted that in the above equation, d, p and t are in mm and f_u is in MPa.

3.2.4 Capacity Factors for the Pull-out Failure of Screwed Connections

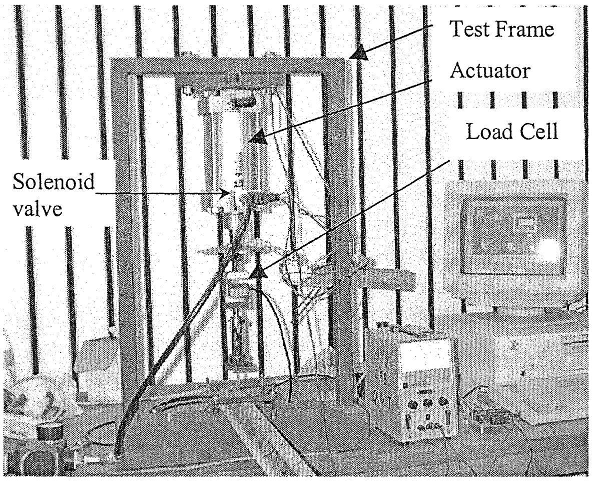
The design equations already in the codes and the proposed equations mentioned in this paper can predict average pull-out strengths based on the limited number of test data. The actual pull-out strength of a real connection can be considerably less than the value predicted by these equations because of the expected variations in material, fabrication and loading effects. Therefore a capacity reduction factor commonly used in design codes should be recommended for the pull-out strength predicted by these equations.

For screwed connections, Pekoz [9] recommended a modified version of the statistical model given in the American cold-formed steel structures code [7] for the determination of capacity reduction factors. This model is used in the Australian cold-formed steel structures code [12]. It was used to calculate the capacity reduction factor ϕ (Table 6). Specified properties were used in the derivation of ϕ factor, and therefore included a correction factor for yield. Measured properties were also used, but both approaches produced approximately the same ϕ factors (Table 6). Further details of these calculations are given in [13,15].

The results clearly indicate that the new simplified design formula has less scatter. The mean Test to Predicted values are more uniform and closer to 1.0 than in other cases. The coefficient of variation is on average less than 0.18 and fairly uniform across different groups whereas the other formulae produced a bigger scatter. Comparison of average and maximum errors for the three formulae confirmed that the new formula produces less errors than other formulae. Based on these observations and previous results, Equation (4) is recommended with a ϕ factor of 0.5. This was possible as the ϕ factors were greater than 0.5 (approximately 0.55). Although steel and screw fasteners used here were obtained from particular manufacturers, results should be equally applicable to other steels and screw fasteners provided they comply with the respective specifications for the steel grades and fasteners used in this investigation.

3.3 Cyclic Tests

A small scale test set-up similar to that used in static tests was used in the cyclic tests, but with constant amplitude cyclic loading conditions as shown in Figure 7. In the static pull-out test series, a larger range of steel grades and thicknesses and screw fasteners was considered. However, in cyclic testing, only a subset of them was considered for two reasons: Fatigue effects were expected to be similar for other combinations of steel battens and screw fasteners; The number of tests may become excessive as at least five cyclic tests had to be conducted for each combination.



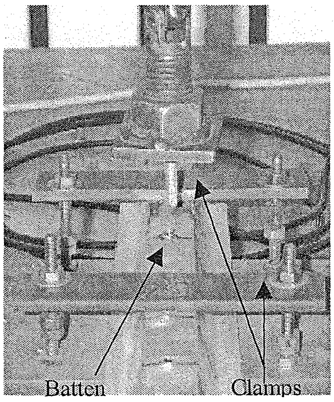


Figure 7. Cyclic Test Set-up

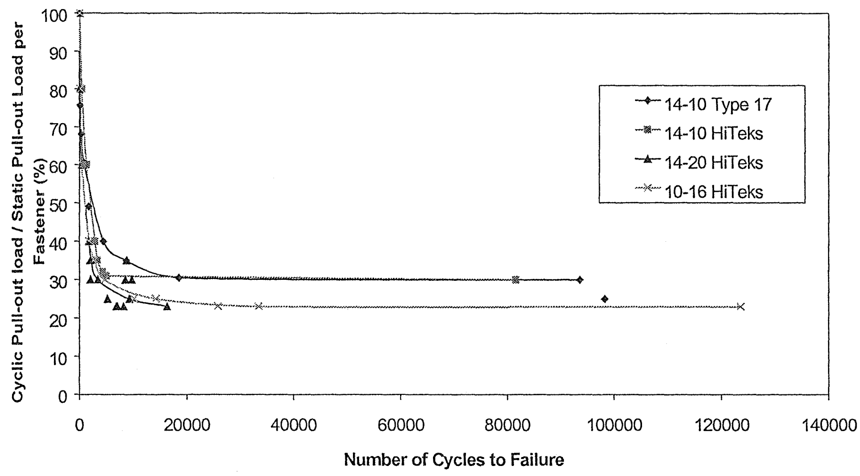
Table 7. Cyclic Test Program

Steel Batten		Screw Fastener		Static Pull-out Failure Load (N/fastener)	Cyclic Load Ranges* as a Percentage of Static Pull-out Failure Load
Steel Grade	Nominal thickness	Type	Gauge		
G550	0.42	Type 17	14-10	1321	25, 30, 30.5, 31, 33, 35, 40, 49, 53, 61, 68, 76
		HiTeks	14-10	1079	30, 31, 32, 35, 40, 60, 80
			14-20	959	23, 25, 30, 35, 40, 60, 80
			10-16	913	23, 25, 30, 35, 40, 60, 80
G550	0.95	Type 17	14-10	3558	20, 25, 30, 35, 40, 50, 60, 70, 75, 80
		HiTeks	14-10	2944	25, 30, 35, 40, 60, 70, 80,
			14-20	2692	25, 30, 35, 40, 50, 60, 80
			10-16	2524	25, 30, 35, 40, 50, 60, 80
G250	0.40	Type 17	14-10	874	35, 37, 40, 50, 60, 80
		HiTeks	14-10	716	30, 35, 40, 50, 60, 80
			14-20	590	40, 50, 60, 80
			10-16	554	60, 80
G250	1.0	Type 17	14-10	2306	30, 35, 40, 50, 60, 80
		HiTeks	14-10	2012	30, 32, 35, 40, 50, 60, 80
			14-20	1800	30, 35, 37, 40, 50, 60, 80
			10-16	1696	30, 35, 37, 40, 60, 80

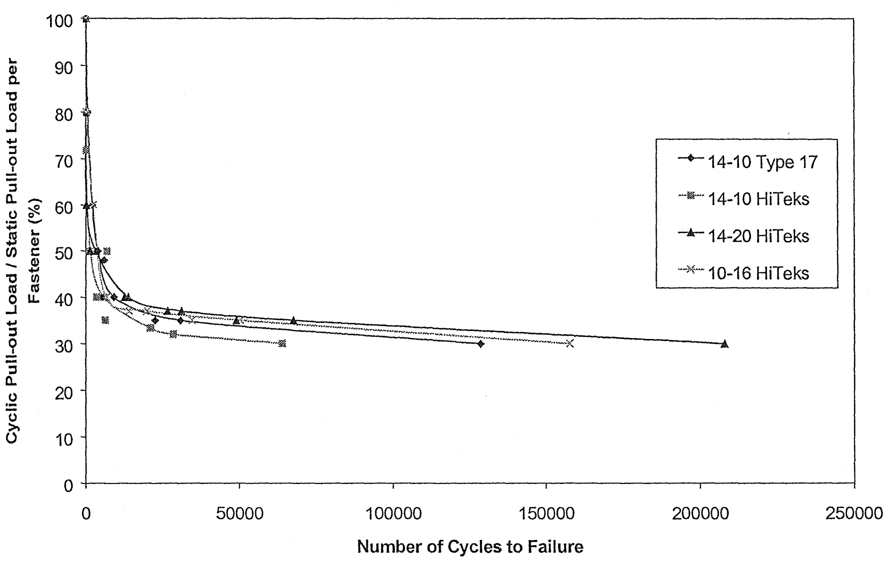
* - Minimum cyclic load = zero

A specially made test frame was used to assemble the test batten and the loading actuator. The test batten was clamped to the base of the test frame at a distance of about 150 mm. As seen in Figure 7, a computer-controlled pneumatic actuator was used to apply the constant amplitude cyclic loading to the screw fastener heads using a special arrangement. These fasteners with a hexagonal head and a neoprene sealing washer were fixed to the test battens in a similar manner to that used in the building industry. Special precautions were taken during the installation process to ensure all screws were centred at the battens, set perpendicular to the plane of the batten and driven inside the batten to a constant length. A series of cyclic pull-out tests was then conducted for a range of combinations of steel battens and screw fasteners until a pull-out failure occurred.

The pneumatic actuator was supplied with compressed air at a regulated pressure. Cyclic loading to the test batten was produced by an air control system in which a process timer operated the actuator. This system was connected to a data acquisition and process control system, which facilitated real time monitoring, integration and processing of test data. The applied load to the screw head was measured by a load cell connected in series with the actuator as shown in Figure 7, and was continuously monitored through a graphic display on the computer. It also had a self-triggering system to stop the system at failure and save the data automatically. By controlling the regulated air supply, the applied cyclic loading was produced at the desired rate. In most of the tests, the loading frequency was maintained at 3 Hz. For each combination of test batten and screw fastener, constant amplitude cyclic load tests were conducted with a load range from about zero to various percentages of its static pull-out load (see Table 7). This resulted in a total of 175 cyclic tests. The cyclic load ranges were based on static test results [13,15], and are included in Table 7. In each test, the cyclic loading was continued until the screw fastener pulled-out from the battens and the corresponding number of cycles was recorded.

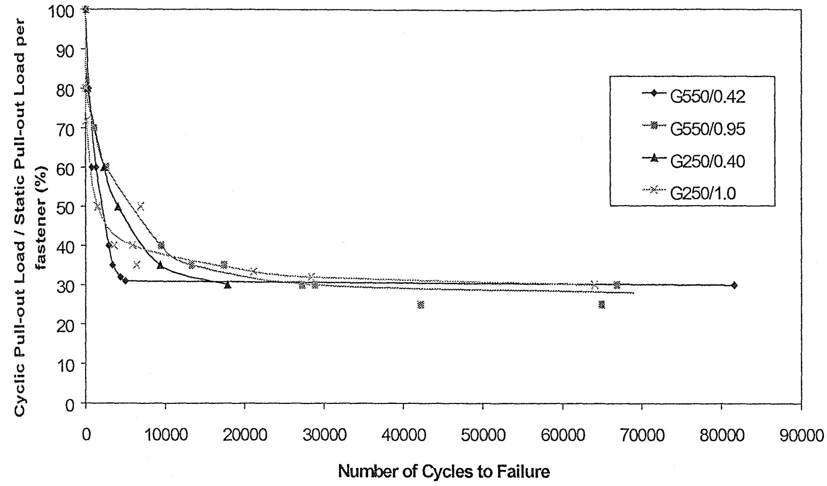


(a) 0.42 mm G550 steel

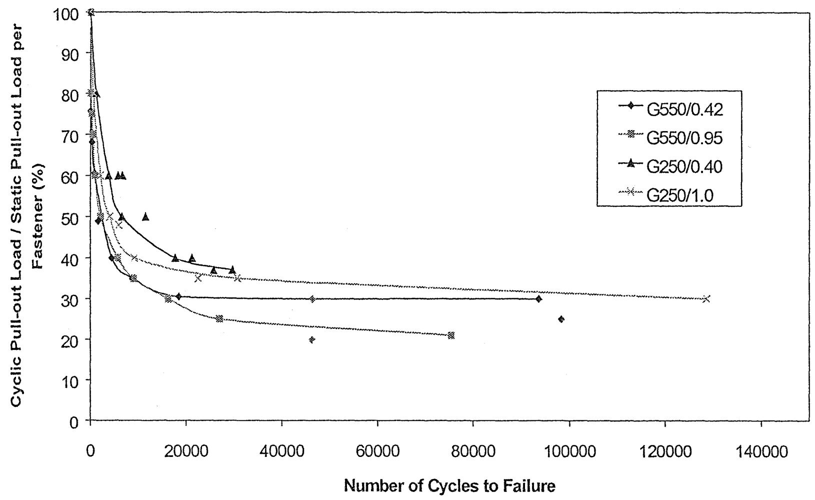


(b) 1 mm G250 steel

Figure 8. Group of Fatigue Curves for Varying Steel and Screw Types



(c) No.14-10 HiTeks Screws



(d) No.14-10 Type 17 Screws

Figure 8. Group of Fatigue Curves for Varying Steel and Screw Types

3.4. Cyclic Test Results and Discussion

3.4.1 Results

Typical experimental results are presented as Cyclic Pull-out failure load (as a percentage of static pull-out failure load per fastener) versus number of cycles to failure in Figures 8 (a) to (d). Other results are presented in [16]. Figures 8 (a) and (b) illustrate the variations in the cyclic behaviour of each steel batten type (steel grade and thickness) due to the use of different screw fasteners whereas Figures 8 (c) and (d) illustrate these variations when different steel batten types are used for the same screw fastener. All the results clearly demonstrate the presence of fatigue effects as the pull-out failures occurred after only a few cycles of loading at much lower load levels than the static pull-out failure loads. In general, there were two modes of cyclic pull-out failure as shown in Figure 9. When the cyclic load was more than about 40 to 50% of the static pull-out failure load, the screw fasteners pulled out as the steel around the fastener holes was bent upwards after a limited number of cycles (< about 10,000) and there weren't any cracking around the fastener holes. The steel bending deformation around the hole was quite small for thicker steel battens. This type of failure was due to the slipping at the connections caused by the upward bending deformations of steel around the fastener hole and cyclic loading. This was particularly true for the thin steel as there wasn't much grip between the fastener and steel. Figure 9 (a) shows the typical failure mode in this case. At higher cyclic loads closer to the static pull-out failure load, the failure was essentially a slipping type failure as for the pure static failures. In summary, the first mode of failure was not an ideal fatigue type failure and occurred after a limited number of cycles. There was a rapid reduction in cyclic pull-out strength in all cases because of this type of failure mode.

(a) Upward Bending and Slipping

(b) Radial Cracking

Figure 9. Cyclic Pull-out Failure Modes

When the cyclic load was less than 40% of the static pull-out failure load, radial cracks appeared around the fastener holes for all grades and thicknesses of steel. These cracks started from the edge of the hole and propagated in all directions. This was due to the repeated deformation that occurs in the vicinity of fastener holes where high stress concentrations were present. Once these cracks propagated sufficiently to let the screw shaft pull-out, the failure occurred suddenly. The above observations were the same irrespective of the steel grade and thickness or the screw type or gauge. Figure 9 (b) shows the typical failure mode observed in this case.

The two contrasting segments of Figures 8(a) to (d) confirm the above discussions about the two types of failure. From these figures, the following observations can also be made.

- Type 17 screw fasteners appeared to give a better cyclic performance for thinner steels. But for thicker steels, no significant difference was observed when different types and sizes of fasteners were used.
- No.10-16 and 14-20 HiTeks screw fasteners appeared to lower the cyclic performance of thinner steels as the combination of smaller pitch and thinner steels did not provide a good resistance against pull-out failures.
- The cyclic performance of steel battens was similar when No.14-10 HiTeks screws were used, however, there were some differences between the different steel thicknesses and grades when other fasteners were used.
- The results from all the connections between the steel battens and screw fasteners considered here appear to indicate the presence of a fatigue limit in the range of 25 to 35% of the static pull-out failure load.

Table 8. Cyclic Test Results

Steel Batten		Screw Fastener		P_{crack}*	Cyclic Load that causes pull-out failure after the following Number of Cycles			
Grade	thickness	Type	Gauge		1000	2500	5000	10000
G550	0.42	Type 17	14-10	x	60	51	40	35
		Hiteks	14-10	x	66	45	31	31
		HiTeks	14-20	x	51	32	29	25
		HiTeks	10-16	x	51	36	30	28
	0.95	Type 17	14-10	x	60	49	42	35
		HiTeks	14-10	x	70	60	50	42
		HiTeks	14-20	40	61	57	51	44
		HiTeks	10-16	40	70	56	48	44
G250	0.4	Type 17	14-10	60	60	50	42	33
		HiTeks	14-10	50	72	59	46	33
		HiTeks	14-20	50	70	57	50	46
	1.0	Type 17	14-10	40	73	58	48	42
		HiTeks	14-10	40	54	46	41	39
		HiTeks	14-20	40	56	52	49	43
		HiTeks	10-16	40	70	60	45	39

* - The amplitude of cyclic load below which fatigue cracks appeared. x – not available

In addition to the results presented in Figures 8 (a) to (d), Table 8 also presents some of the results from the cyclic tests. It includes the loads below which the pull-out failure associated with fatigue cracking occurred. These loads indicate that this load is in the range of 40-50% of the static pull-out failure load. Table 8 also includes the level of cyclic load that caused a pull-out failure after a specified number of cycles as obtained from the fatigue curves. The cyclic load is expressed as a percentage of static pull-out failure load.

The design for cyclone wind loading conditions in Australia requires that the steel roof cladding systems pass a three-level low-high fatigue test sequence [17]. The three-level low-high fatigue test sequence includes the following loading: 8,000 cycles at 0 to 0.4 x ultimate design load (F_u), 2,000 cycles at 0 to 0.5 F_u and 200 cycles at 0 to 0.6 F_u. However, the design for the Northern Territory in Australia requires a more severe loading sequence made of 10,000 cycles at 0 to 0.67 F_u. These fatigue test sequences are considered to simulate cyclone wind load conditions on roofing systems. The results given in Table 8 can therefore be used by designers to determine the design pull-out failure load for cyclone wind loading conditions depending on the screw fastener and steel batten used. For multi-level fatigue test sequences, the use of an appropriate fatigue damage rule such as Miner's law is required to estimate the design pull-out failure load for cyclone wind conditions.

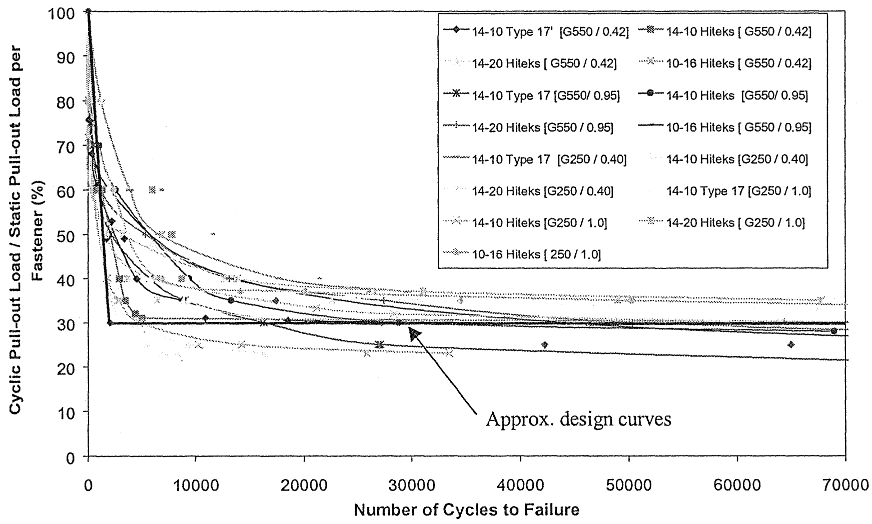


Figure 9. Fatigue Curves

10

3.4.2 Design Method

Although the results in Section 3.4.1 can be used directly by designers of steel roof and wall cladding systems, it is important that a simpler design method is developed to take into account the significant reduction to the pull-out strength caused by cyclic wind loading. For this purpose, all the cyclic test results obtained from this investigation were plotted in the same figure (Figure 9), and simple design equations (Equation (4)) shown next were obtained as an approximate lower bound. These equations give the necessary reduction factor R (cyclic pull-out strength to static pull-out strength) as a function of the number of loading cycles N.

For $N \leq 2000$, $R = 1 - 0.70\,(N/2000)$ (4a)
For $N > 2000$, $R = 0.30$ (4b)

These equations can be used for design wind events with only one load level, for example, the fatigue loading sequence used in the Northern Territory to simulate cyclonic loading. Equation (4b) is conservative for almost all cases whereas Equation (4a) may be unconservative in some cases. However, the combination of these two equations is expected to provide conservative results for all types of connections. It is recommended that No.10-16 and No.14-20 screw fasteners are not used with thinner steels (0.40 and 0.42 mm), in which case, the applicability of recommended equations will not be limited.

The simple design equations may be considered conservative as they were based on an approximate lower bound to all the test results. However, it can be improved by developing similar equations, but which are specific for a given combination of steel and fastener types based on its fatigue curves such as those shown in Figures 8 (a) to (d). The results given in Table 8 can also be used instead of the fatigue curves.

For a design wind event with a wind loading spectrum with more than one load level, these simple equations can still be used in determining the design pull-out load more accurately, provided a fatigue damage law such as Miner's law is used. It is not known whether the use of Miner's law based on a linear cumulative damage model is adequate to determine the total fatigue damage caused by a wind loading spectrum. However, a simpler, but more conservative design approach based on the observed fatigue limit can be used. Since this investigation indicated the presence of a fatigue limit of about 25 to 35% of the static pull-out failure load, it is recommended that a reduction factor of 0.3 can be used in the design of steel cladding systems to allow for the effects of wind loading fluctuations on pull-out strength.

In order to investigate the use of Miner's law, Mahendran and Mahaarachchi [16] conducted a series of multi-level cyclic tests based on the three-level loading sequence recommended by the Australian wind loading code [17]. Their results indicated that the type of load sequence has only a minor effect on fatigue damage and that the results are similar for both steel grades. The results also indicate that Miner's law underestimates the fatigue damage (<1.0). Therefore it is recommended that Miner's law based on a simple cumulative fatigue theory can be used to predict the design pull-out failure load more accurately for a given wind event with multiple loading regimes (eg. cyclone/storm conditions) provided it is modified by a factor of 0.7.

4. Conclusions

An experimental investigation involving a large number of static and cyclic tests (>800) has been conducted on connections between steel battens made of different thicknesses and steel grades, and screw fasteners with varying diameter and pitch under wind uplift/suction loading.

Analysis of the static test results showed that the current design formula for the pull-out strength may not be suitable for the screw fasteners and the thin high strength steels considered in this investigation. This design formula gave conservative results only for thicker ($1.5<t\leq3.0$ mm), softer grade steels. A simple design formula that models the pull-out failure more accurately has been developed for the battens, purlins and girts used in the building industry. This formula has been developed in terms of not only the thickness and ultimate tensile strength of steel and the thread diameter of the screw fasteners, but also the pitch of screw fasteners. For this improved formula a capacity reduction factor of 0.5 as given in the American and the Australian Cold-formed Steel Structures codes was found to be acceptable. Cyclic test results revealed the significant reduction to pull-out strength caused by fluctuating wind loading. Simple design equations and suitable recommendations have been made. This paper has presented the details of the investigations and the results.

Two research reports [15, 16] have presented raw test data and further details that can be used for other purposes.

5. Acknowledgements

The author wishes to thank two of his postgraduate research scholars, Louis Tang and Dhammika Mahaarachchi, who conducted the tests and analyses reported in this paper, BHP Sheet and Coil Products, Stramit Industries, and ITW Construction Products for the donations of experimental materials, Queensland University of Technology (QUT) for its financial support for this research project through the ARC Small Grants Scheme and QUT's Physical Infrastructure Centre and School of Civil Engineering for providing the required research facilities and support.

6. References

1. Mahendran, M. (1994), Behaviour and Design of Profiled Steel Roof Claddings under High Wind Forces, Engineering. Structures, 16 (5), 368-376.

2. Beck, V.R. and Stevens, L.K. (1979), Wind Loading Failures of Corrugated Roof Cladding, Civil Eng. Trans,. I. E. Aust., 21 (1) 45-56.

3. Mahendran, M. (1990a), Fatigue Behaviour of Corrugated Roofing under Cyclic Wind Loading, Civil Eng. Trans., I.E.Aust., 32 (4), 219-226

4. Mahendran, M. (1990b), Static Behaviour of Corrugated Roofing under Simulated Wind Loading, Civil Eng. Trans., I.E.Aust., 32 (4), 211-218

5. Xu, Y.L. and Reardon, G.F. (1994) Tests of Screw Fastened Profiled Roofing Sheets Subject to Simulated Wind Uplift. Engineering Structures, 15 (6): 423-430.
6. Standards Australia (SA) (1996), AS/NZS4600 Cold-Formed Steel Structures Code.
7. American Iron and Steel Institute (AISI) (1989), Specification for the Design of Cold-formed Steel Structural Members, 1986 Edition with 1989 Addendum, AISI, Washington, DC.
8. Eurocode 3 (1992), Design of Steel Structures, Part 1.3 - Cold-formed Thin-gauge Members and Sheeting, Draft document CEN/ TC250/SC3 - PT1A, Commission of the European Communities, August.
9. Pekoz, T. (1990), Design of Cold-formed Steel Screw Connections, Proc. of the Tenth International Speciality Conference on Cold-formed Steel Structures, St. Louis, Missouri, USA, 576-587
10. Toma, A., Sedlacek, G. and Weynand, K. (1993), Connections in Cold-formed Steel, Thin-walled Structures, 16, 219-237.
11. Macindoe, L. and Hanks, P. (1994), Standard Tests for Cold-formed Steel Single Fastener Connections, Proc. of Australasian Structural Engineering Conference, Sydney, 253-257.
12. Macindoe, L. Adams, J. and Pham, L. (1995), Performance of Single Point Fasteners - Report to the CRC for Materials, Welding and Joining, CSIRO Division of Building, Construction and Engineering, Melbourne.
13. Mahendran, M. and Tang, R.B. (1998) Pull-out Strength of Steel Roof and Wall Cladding Systems. ASCE Journal of Structural Engineering: 124 (10): 1192-1201.
14. IITW Construction Products (1995), The Buildex Screw Fastener Book, Melbourne.
15. Mahendran, M. and Tang, R.B. (1996) Pull-out Strength of Steel Roof and Wall Cladding Systems, Research Report 96-38, Physical Infrastructure Centre, Queensland University of Technology, Brisbane, Australia
16. Mahendran, M. and Mahaarachchi, D. (2000) Cyclic Pull-out Strength of Steel Roof and Wall Cladding Systems, Research Monograph 2000-10, Physical Infrastructure Centre, Queensland University of Technology, Brisbane, Australia.
17. Standards Australia (SA) (1989) AS 1170. Loading Code Part 2: Wind Loads, Sydney.